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Featuring

Discussion of Report of Joint Committee on Concrete and Reinforced Concrete

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Volume XII FEBRUARY, 1925 Number 2

DISCUSSION OF THE "REPORT OF THE JOINT COMMITTEE ON STANDARD SPECIFICATIONS FOR CONCRETE AND REINFORCED CONCRETE" SUBMITTED TO CONSTITUENT ORGANIZATIONS, AUGUST 14, 1924*

(Presented before the Designers Section, B. S. C. E., on December 10, 1924)

THE MAKING OF CONCRETE

(Covering Chapters I to X of the Report)

By Dean Peabody, Jr., Member, Boston Society of Civil Engineers†

The papers that follow describe the changes it is proposed to make n the accepted practice of design of reinforced concrete structures. The best of design is of little use, however, if the concrete does not give the strength assumed in the specifications. Such a failure may be due to poor materials, faulty proportions, errors in mixing or in placing. It is the province of the first ten chapters of the Joint Committee's Report to specify methods of procedure, which, if rigorously followed, will ensure good concrete in place in the structure.

The contents of the last report of the affiliating societies, adopted in 1916, are familiar to all civil engineers. This was in the form of "Recommended Practice." In 1921 "Tentative Specifications" appeared, and now in 1924 "Standard Specifications for Concrete and Reinforced Concrete." Since 1916 a wealth of test data on debatable points has accumulated. Reports have been printed on the strength of concrete, ts modulus of elasticity, its wear, on the amount of water to use, on the use of admixtures, on the use of slag as an aggregate, on integral waterproofing, and on the methods of proportioning the mix. My topic will be the advance since 1916 in the accepted methods of making concrete.

---

* The Joint Committee on Standard Specifications for Concrete and Reinforced Concrete consists of five representatives from each of the following: American Society of Civil Engineers, American Society of Testing Materials, American Railway Engineering Association, American Concrete Institute, Portland Cement Association. The report, submitting standard specifications for concrete and reinforced concrete, s printed in the Proceedings for October, 1924, of the American Society of Civil Engineers, and also in publications issued by the other constituent organizations.

† Assistant Professor of Applied Mechanics, Massachusetts Institute of Technology, Cambridge, Mass.

Quality of Concrete

If one assumes a certain strength in design, the obvious procedure is to test the actual mix to see if it gives at least that strength. If it does, no other test is needed. It has not been easy to require the obvious thing. There is the extra work of taking specimens of concrete in the field, and of storing, shipping and testing them. There are disputes in regard to the assessment of the cost of testing; and, above all, there is the problem of what to do, if, at the end of twenty-eight days, concrete, which is now an integral part of the structure, is found to be too weak.

Materials

The materials of concrete are cement, aggregate, and water. The selection of Portland cement has long been standardized and is, for the most part, carefully attended to. The standardization of the tests for the selection of the aggregate seems to have been completed now.

The tests for the fine aggregate, or sand, include the screening, or mechanical analysis, of the material, enabling one to figure the fineness modulus, or grading constant. Tests are made for the tensile or compressive strength of the mortar in comparison with the Ottawa sand. It is interesting to note that sands are usually rejected which are less than 100% of the Ottawa; while 70% was regarded as the minimum in 1916. The colorimetric test for organic matter is also required.

Coarse aggregates are inspected for appearance and a sieve analysis is made. The fight for the use of blast furnace slag is won, as it is not now prohibited.

The water shall be clean and free from injurious amounts of oil, acid, alkali, organic matter, or other deleterious substance. The consistency of the concrete mix is determined by the slump, or cone, test. Concrete is rammed into a truncated cone, the cone is removed, and the consistency recorded as inches of slump. Ordinary mixes give three to six inches of slump.

Proportioning Concrete

If the materials are desirable, the most important problem of all can be approached. This is to determine the most economical proportions of the mix; or the mix giving the desired strength with the least amount of cement. In 1916 it was recommended that the aggregates be so chosen that the resulting concrete should have the maximum density, or minimum voids. Since then Professor Abrams has advocated proportioning by "fineness modulus and water ratio"; Messrs. Edwards and Young have proposed proportioning by "surface areas", as one would paint; and further confirmation of maximum density proportioning has been given by Professor Talbot of Illinois and Mr. Crum of the Iowa Highway Division. Abrams's " water ratio," Young's " surface areas", and Crum's "density" methods have all been used on good-sized concrete jobs. All these methods produce as a result dense, strong concrete. The method of proportioning adopted in the 1921 and 1924 reports is that of "fineness modulus" and "water ratio." The tables in Appendix XVI of the Joint Committee's Report are based on several thousand laboratory tests and on some years of use in the field. Local conditions vary so much that the strengths given are approximate only, but this table gives the relative variation of the strength as different sizes of fine and course aggregates are used, and as different degrees of wetness of mix are needed. Of course no more water will be used than is necessary to give the required "mobility". It may seem strange, at first, to specify fractional parts to a mix, such as a 1 : 2.1 : 5.8 mix; but, after all, any mix is merely a matter of the relative size of the measuring boxes, or of the weights on the scale beam.

One of the hottest debates of the 1921 "Tentative" report has been on the three options for specifying the proportions of the mix. Option 1 gave a specified mix based on tests of the materials to be used. Option 2 gave the mix by use of the strength tables of Appendix XVI. This assumed that the materials used were of average quality. Option 3 left the mix to the contractor, and required only that his concrete should be tested, and satisfactorily meet the specified strength. The determination of the most economical mix was thus left to the initiative of the contractor. These field tests are to be paid for by the owner. All three of these options are in the present report. No provision has been made covering the case where the concrete fails to meet the required strength.

Mixing

The general principles governing the mixing of good concrete have long been known. Since 1916 careful tests on concrete mixers at different speeds, with varying times of mixing and with different sizes of charge, have been made. The only change in the report is to reduce the minimum time of mixing from 1½ and 2 minutes to 1 minute. In hand mixing, the number of complete turns of the entire mass has been reduced from six to three.

DEPOSITING CONCRETE

There has been little change in the instructions for depositing co crete. The 1916 Report did not recommend the use of salt in freezin weather. Salt, accelerators, and other admixtures are now left to th discretion of the engineer in charge. There are many tests in print no of the effect of admixtures on concrete strength to assist the engineer i making his decision.

PROTECTIVE COVERINGS

Tests for fire resistance have shown us the necessity of careful sele tion of aggregates, particularly for columns. Limestones and traps giv the best results. Quartz rocks are to be avoided. The tendency is t provide somewhat greater protection for the steel.

WATERPROOFING

In 1916 integral waterproofing compounds were prohibited. Th ensuing debate has resulted in reference of the decision to the engine in charge.

CONCLUSION

Whether every provision for the selection and proportioning the concrete shall be used depends on the type of the job. Small jobs light loads do not warrant much refinement. One must determine f each case how far to go; but a careful following of the full procedure ca only result in a concrete, well made, well placed, sufficient for its load, an a delight to behold.

BEAMS AND COLUMNS

(Covering Parts A, B, C and G, Chapter XI of the Report)

BY CHARLES E. NICHOLS, MEMBER, BOSTON SOCIETY OF CIVIL ENGINEERS*

PART A — GENERAL ASSUMPTIONS

The general design assumptions (except as mentioned later und column stresses) are the same as in the old Joint Committee Repor except for slight changes in the moduli of elasticity of concrete. Whe previously values of 1/40 the modulus of elasticity of steel for concre of less than 800 lb. per sq. in. ultimate strength, and 1/12 for 800 lb. per sq. in. to 2,200 lb. per sq. in. were used, the new report gives 1/12 for concrete between 1,500 and 2,200 lb. per sq. in. and does not specify a value for concrete of less than 1,500 lb. strength.

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* Engineer, Stone & Webster, Inc., Boston, Mass.

PART B — FLEXURE OF RECTANGULAR REINFORCED CONCRETE BEAMS AND SLABS

Formulas for computing flexure, including the lengths of spans to be used, are the same as previously recommended. Slight changes have been made in the moment coefficients to be used for beams and slabs built to act integrally with several restraining supports. The accompanying chart shows a comparison of the old and new specifications for three general cases: (*a*) nonrestraining or simple supports; (*b*) partially restraining supports; and (*c*) fully restraining or rigid supports. Where the old recommendations were not specific the chart shows the writer's interpretation of its intent. Coefficients from the old report are shown above the beams and from the new report below. (See Fig. 1.)

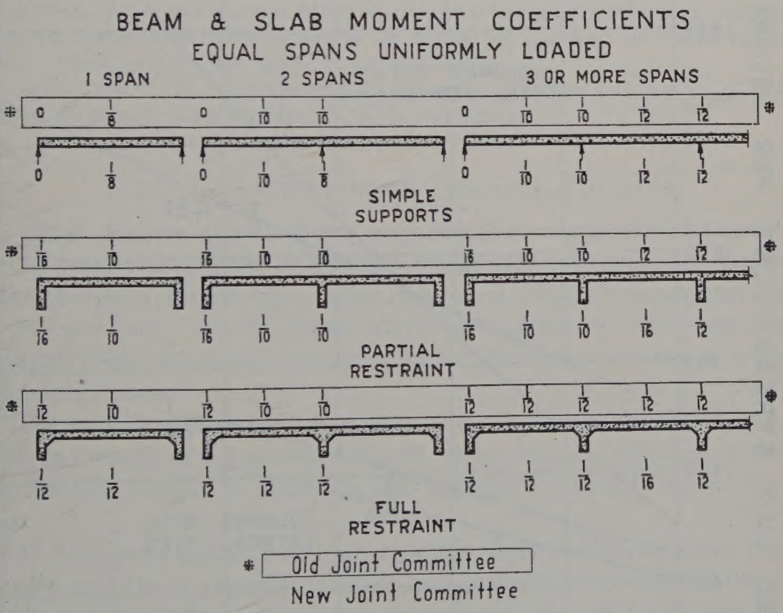

FIG. 1

In general these changes show a greater tendency to follow the lines

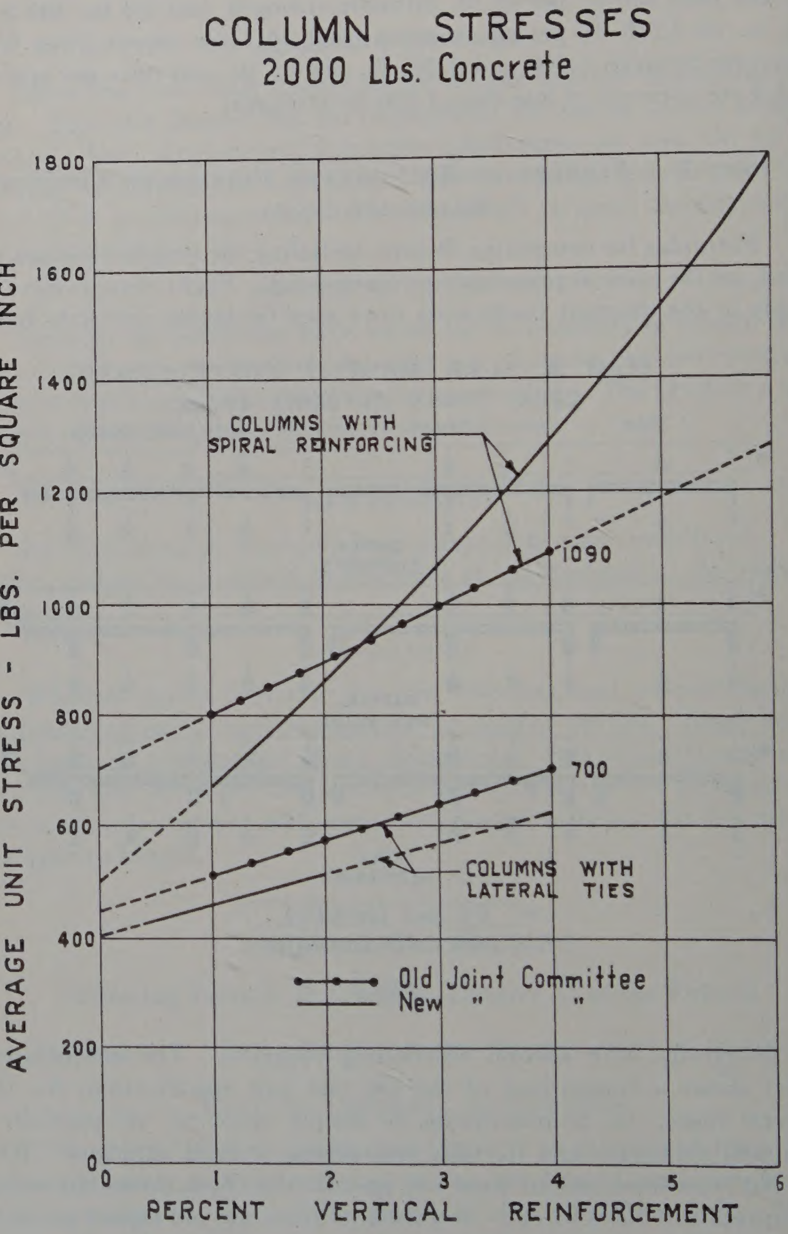
COLUMN STRESSES
2000 Lbs. Concrete
AVERAGE UNIT STRESS - LBS. PER SQUARE INCH
1800
1600
1400
1200
1000
800
600
400
200
0
0
1
2
3
4
5
6
PERCENT VERTICAL REINFORCEMENT
COLUMNS WITH SPIRAL REINFORCING
1090
700
COLUMNS WITH LATERAL TIES
Old Joint Committee
New " "

FIG. 2

of rigid frame analyses, which the old Joint Committee perhaps lacked courage to recommend eight years ago.

The new report requires the width of beam compression flanges to be not less than 1/24 the distance between lateral supports.

PART C — FLEXURE OF REINFORCED CONCRETE T-BEAMS

This portion of the specification is essentially in agreement with the old report. Minor changes are as follows:

Section 115. — Permits a compression flange widtn of eight (instead of six) times the slab thickness each side of the web.

Section 116. — For slabs in which the main steel parallels the beam, a minimum of 0.3 per cent transverse reinforcement in the top of the slab over the beams is now required. The old report was indefinite on this point.

Section 119. — The requirements for proportions of the slab flanges of isolated T-beams are slightly more liberal than in the old report.

PART G — REINFORCED CONCRETE COLUMNS

While the new report shows very few and only minor changes from the old report in regard to general design assumptions and flexure in beams and slabs, the changes in the reinforced column stresses and details are very marked. Briefly, these changes are such as to encourage the use of columns with spiral reinforcement and to discourage the use of so-called "rodded columns."

The specifications are based upon short columns for which the unsupported length is not greater than 40 times the least radius of gyration. This limit of length is about 2/3 the permissible length under the old report.

The accompanying chart shows clearly the relations between the old and new reports as regards average unit stresses for axial compression, both in columns with vertical rods and closely spaced hoops and in columns with vertical rods and lateral ties or binders. The full lines represent the stresses within the permitted limits of minimum and maximum reinforcement, and these lines have been extended (broken lines) on the chart for purposes of comparison. (See Fig. 2.)

Perhaps the most startling fact shown by this chart is that a spirally reinforced column with 6% vertical steel and 1½% hooping may be expected to support safely a load giving an average unit stress equal to the ultimate compressive stress of the concrete alone.

The new specification for spiral columns differs radically from the old. The unit stress allowed on the concrete core is a function of the per cent of the vertical reinforcement used and as such is only indirectly a function of the per cent of hooping. From 1% to 6% vertical steel may be used as compared with a 4% maximum previously, and the hooping is now specified as 25% of the vertical steel, or from ¼% to 1½% of the concrete core. At least 1% spiral steel was formerly required. For percentages of vertical steel below about 2½% (or below 0.6% hooping) the average total stresses are lower than those permitted by the old report and tend to approach the stresses allowed on columns with lateral ties or binders. For the higher percentages of steel, however, the total stresses are now considerably higher and the maximum is nearly double the former maximum.

Columns without hooping are now limited to a maximum of 2% vertical steel and a minimum of ½%, as compared with previous limits of 4% and 1% respectively. Closely spaced binders are now required, at least ¼″ rods 8″ apart. The working unit stress has also been reduced about 10%, being now 0.20 the ultimate strength instead of 0.225.

The new specification, however, permits figuring the gross column section as carrying load and does not, except for very hazardous conditions, require deducting the outer 1½″ of the column, as fireproofing only. This results, under normal exposures to fire, in greater allowable loads on columns smaller than 20″ square and smaller loads on larger columns.

The new report specifies allowable stresses under combined bending and axial compression of 20% more than direct compression in spiral columns, and 50% more in columns with lateral ties, with 4% instead of 2% maximum permissible steel.

In the method of designing columns with cast iron on structural steel cores the new specification departs from the general assumptions that initial stresses in steel due to shrinkage of concrete shall be neglected, and that steel and concrete stresses will be in proportion to their relative moduli of elasticity. For such columns (when reinforced with hooping and vertical bars) the cores may be designed essentially as ordinary steel or cast-iron columns, and the concrete also allowed to take load with a stress of 0.25 the ultimate strength. This method is based upon the results of column tests with which the average practicing engineer is perhaps not familiar.

It is not entirely clear why this consideration of initial steel stress due to concrete shrinkage should not also apply to the other types of reinforced columns. However, in the case of columns with lateral ties the resulting increase in total allowable load would not be considerable and perhaps this factor may indirectly be the cause of the comparatively high stresses now permitted for spiral columns.

A definite formula is now given for reducing the column working stresses for columns longer than 40 times the least radius of gyration. This was not covered in a definite way in the old Joint Committee Report.

DIAGONAL TENSION AND SHEAR IN BEAMS

(Covering Part D, Chapter XI of the Report)

By Hale Sutherland, Member, Boston Society of Civil Engineers*

The section on diagonal tension and shear opens with a statement of the notation used, followed by the usual formula for determining the shearing unit stress in reinforced concrete beams. In applying this formula to the joists in beam-and-tile construction it is specified that the width of the effective section for shear, as governing diagonal tension, be taken as the thickness of the web plus one-half the thickness of the vertical webs of the tile in contact with the beam.

A striking change from previous reports is the recommendation that in the design of beams carrying uniform loads not less than one-fourth as much shearing resistance be provided at the point of zero shear as at the end of the span, thus calling our attention to the fact that a portion of the uniform load for which we design our beams in building construction is movable load and therefore will cause shear and diagonal tension at the center, where custom has always assumed the shear to be zero except in the case of large and important beams. Accordingly, in computing the diagonal tension in a beam carrying a uniform load on its entire length the shear curve is to be taken as a straight line from the maximum value at the end to one-fourth the maximum at the center. This requirement does not increase the number of stirrups, as would at first appear, since the 1916 Joint Committee recommendation that the steel reinforcement be designed to carry two-thirds of the diagonal tension is dropped, and it is permitted to figure on the concrete as carrying all of the diagonal tension measured by a unit shear intensity of 2% of the ultimate compressive strength of the concrete ($0.02f'_c$).

Another admirable change from the 1916 Report (the one now standard by vote of the several societies concerned) is a simplification regarding beam types as based on web reinforcement. The distinction based on the secure attachment of the web members to the horizontal steel is not repeated. In the present report we have these types: (*a*) beams without web reinforcement, with intensity of unit shear limited to $0.02f'_c$ where the longitudinal bars are without special anchorage, and to $0.03f'_c$ with longitudinal bars having special anchorage; (*b*) beams with web reinforcement consisting of a series of stirrups, vertical or inclined, or of a series of bent-up longitudinal bars, or of a combination of stirrups and bent-up bars, the shear being limited to $0.06f'_c$ where the longitudinal bars are without special anchorage, and to $0.12f'_c$ with longitudinal bars having special anchorage; (*c*) beams with web reinforcement consisting of longitudinal bars bent up at a single point with unit shear intensity limited to 75 lbs. per sq. in. plus either $0.03f'_c$ or $0.02f'_c$, according to whether the longitudinal steel is with or without special anchorage.

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* Assistant Professor of Structural Engineering, Massachusetts Institute of Technology, Cambridge, Mass.

This last type of web reinforcing was recommended by Mr. Godfrey, a member of the 1916 Joint Committee, in a footnote at the end of their report, and was formally recommended in the 1921 Report with instructions to compute the stress (S) in these bent rods as though the vertical component of that stress were the portion of the maximum end shear (V′) assumed to measure the diagonal tension carried by the steel ($S = V' + \sin a$—where "a" is the angle between the bent rods and the longitudinal steel), a method of analyis that differs radically from that usually employed. No direct instructions are given in the 1924 Report for determining the stress either in this last type of web reinforcement or in any of the others.

This omission of formulas giving directly the stress in stirrups and bent rods is curious since two formulas are given for the determination of the limits of shearing unit stress each of which contains a term that indicates how it is intended the stress in web reinforcement should be computed, and the two terms apparently are not in agreement.

The first of these formulas for allowable unit shear stress (v) applies where web reinforcement slopes at an angle "a" of from 45° to 90° with the longitudinal steel:

$$v = 0.02f'_c + \frac{f_v A_v}{bs \sin a} \quad \ldots . \text{(31) sect. 127.}$$

In addition to the notation already given the following symbols are used:

f_v = tensile unit stress in web reinforcement.
A_v = total area of web reinforcement within a distance, s.
b = beam width.
s = longitudinal spacing of web bars or stirrups.

With the constant above given this formula for v applies to beams without special anchorage of longitudinal reinforcement and the limiting value of v is set at $0.06f'_c$. If special anchorage is given to the longitudinal steel $0.03f'_c$ is substituted for $0.02f'_c$ and the limiting value of v is raised to $0.12f'_c$.

In words the above formula says that the total shearing unit stress at any point equals that measuring the diagonal tension carried by the concrete v_c plus that measuring the diagonal tension carried by the steel v'. Inspection of the second term of the equation shows it to be equal to v' when the stress in the web member is assumed to be the component of the diagonal tension parallel with it, *the other component being taken at right angles to the bar* (see explanatory notes). This is the method adopted by this Joint Committee in their 1921 Report, their formulas for stress S in web members being --

$$S = \frac{V's}{jd} \quad \ldots . \text{ (30) 1921 Report — for vertical stirrups.}$$

$$S = \frac{V's}{jd} \sin a \quad \ldots . \text{ (31) 1921 Report — for inclined bars.}$$

Here V' equals the "external shear on any section after deducting that carried by the concrete", and jd is the lever arm of the resisting couple.

The second formula applies for slopes between 15° and 45° and equals —

$$v = 0.02f'_c + \frac{f_v A_v}{bs} (\sin a + \cos a) \quad \ldots . \text{ (32) sect. 127}$$

with the same limits as the first formula (31). The second term of this expression equals v' (the unit shear intensity measuring the diagonal tension in the steel), when the stress in the inclined rod is assumed to be the component of diagonal tension parallel to it, *the other component being taken parallel to the longitudinal steel* (see explanatory notes). For a slope of 45° these two formulas give equal results, but they differ widely for other angles, as is shown by the curves in Fig. 3.

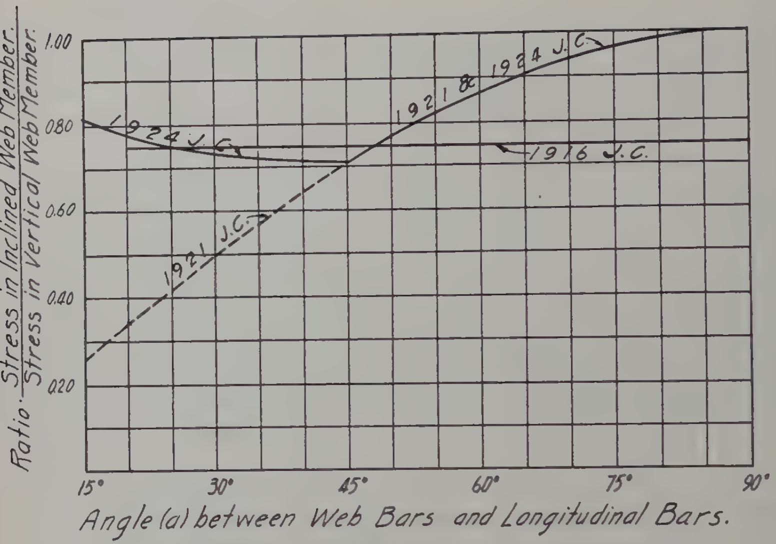

FIG. 3. STRESS (S) IN INCLINED WEB REINFORCEMENT

1916 Joint Committee ($a=20°-90°$) . . .	$S=\frac{3}{4}\frac{V's}{jd}$
1921 Joint Committee ($a=20°-90°$) 1924 Joint Committee ($a=45°-90°$) . . .	$S=\frac{V's}{jd}\sin a$
1924 Joint Committee ($a=15°-45°$) . . .	$S=\frac{V's}{jd\,(\sin a+\cos a)}$

The 1916 Joint Committee recommended that the stress in an inclined web member be taken as three-fourths of that in equivalent vertical web reinforcement, the minimum slope being set at 20° instead of at 15° as in the present report.

In all discussions of diagonal tension these words from the 1916 Report of the Joint Committee should be kept in mind:

> In designing, resource is had to the use of calculated vertical shearing stresses as a means of comparing or measuring the diagonal tension stresses developed, it being understood that the vertical shearing stress is not the numerical equivalent of the diagonal tensile stress, and that there is not even a constant ratio between them. — It does not seem feasible to make a complete analysis of the action of web reinforcement and more or less empirical methods of calculation are therefore employed.

In view of these considerations the advisability of using these two cumbersome formulas is much to be doubted. The 1916 ruling that the stress in the inclined bar be taken as three-fourths of the stress in equivalent vertical reinforcement would seem to be as accurate an estimate as we are warranted in making. However, the Committee is to be commended for being more conservative than in 1921 as regards the flatter slopes.

The maximum longitudinal spacing of web reinforcement of all slopes is limited by the following formula in the 1924 Report:

$$s = \frac{45\ d}{a+10}$$

"d" being the depth of the beam to the longitudinal steel. In the 1916 Report the maximum spacing for vertical stirrups is d/2; and for inclined bars 3/4d. Thus the present Committee is slightly more conservative than that of 1916 as regards vertical stirrups and much more radical as regards inclined web reinforcement.

In the matter of the use of two or more types of web reinforcement the present report is more simple and direct than that of 1916, each element being assumed to carry its share of the load in accordance with the methods we have been reviewing.

Thus it appears that this section on diagonal tension shows advance in some respects over previous Joint Committee recommendations, and in others a tendency toward complicated analyses of doubtful worth that require explanation before they can be accepted as standards for our practice.

Explanatory Notes

This expression $\frac{f_vA_v}{bs \sin a}$ may be derived in this way. In Fig. 4 is shown one end of a reinforced concrete beam with inclined bars crossing a 45° line of diagonal tension rupture. Evidently the portion of the diagonal tension taken by the middle rod is that acting in a length "m" along the crack, and equals $v'bm$, using the notation already familiar. If it be assumed that the sloping bar carries one component of this stress, the other component being taken perpendicular to the steel, we have for the stress (S) in the reinforcement (Fig. 4)

$$\begin{aligned} S &= v'\ bm \cos (45^\circ - a) \\ &= v'bs \sin a \\ &= \frac{V's}{jd} \sin a \quad \text{since } v' = \frac{V'}{bjd} \end{aligned}$$

Accordingly, $v' = S \div bs \sin a = f_vA_v \div bs \sin a$ as in the formula quoted, (31) sect. 127, 1924 Report.

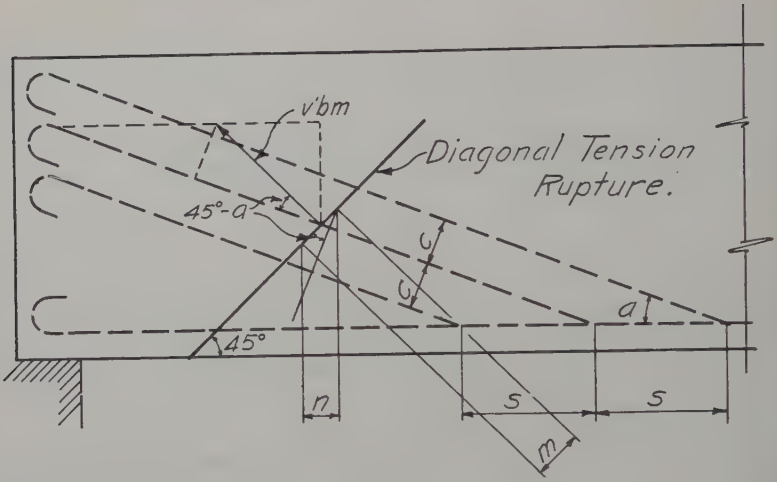

FIG. 4

$$c = s \sin a$$
$$m = c \div \cos(45° - a) = \frac{s \sin a}{\cos(45° - a)}$$

$$n = m \cos 45° = \frac{s \sin a \cos 45°}{\cos 45° \cos a + \sin 45° \sin a} = \frac{s \sin a}{\cos a + \sin a}$$

A different solution of the problem is suggested by Turneaure and Maurer in "Principles of Reinforced Concrete Construction". The vertical component of the diagonal tension, v'bm, is v'bn. If this be taken as the vertical component of stress in the reinforcement we obtain —

$$\begin{aligned} S &= v'bn \div \sin a \\ &= v'bs \div (\cos a + \sin a) \text{ (see Fig. 4)} \end{aligned}$$

Accordingly —

$$\begin{aligned} v' &= S (\cos a + \sin a) \div bs \\ &= f_v A_v (\cos a + \sin a) \div bs \end{aligned}$$

as in the second formula quoted, (32) sect. 127, 1924 Report. This method of resolving the diagonal tension is equivalent to making one component parallel to the inclined steel and the other parallel to the longitudinal steel. The writer is indebted to Mr. H. A. Gray for calling his attention to the formula of Turneaure and Maurer as explaining the second formula (32).

The following interpretation of the first and more common theory makes plain how radical is the 1921 recommendation regarding stress in inclined web reinforcement. The expression $S = v'bs \sin a$ in words says that the stress in the web reinforcement equals the vertical component of diagonal tension in the distances s multiplied by sin a. This is equivalent to assuming that the horizontal component of diagonal tension is carried by the horizontal steel and the vertical component by vertical stirrups ($a = 90°$) or by inclined rods and the concrete, the vertical component being resolved into a component parallel to the steel and one perpendicular to it. The component at right angles to the reinforcement would be offset by the diagonal compression. Plainly the flatter the slope of the inclined steel the less its stress and the less effective would be the diagonal compression in resisting the other component.

BOND AND ANCHORAGE IN BEAMS

(Covering Part E, Chapter XI of the Report)

By Walter W. Clifford, Member, Boston Society of Civil Engineers*

The section on Bond and Anchorage starts with the familiar formula $u = \frac{V}{\Sigma ojd}$ for bond stress on beam reinforcement. It goes on to define the points at which the bond must be computed, namely, at the point of inflection for positive steel, and at the support and the point of inflection for negative steel in restrained or continuous beams, and at the face of the support for simple beams. A new feature is the proviso that bent-up rods which are within the distance of d/3 from the horizontal steel at a critical section may be included in figuring Σo. This would apparently affect computations at the point of inflection only, for ordinary beams, and does not seem of great significance.

The allowable stresses appear to be the same as those we have been using, namely, $0.04f'_c$ for plain bars and $0.05f'_c$ for deformed bars with "ordinary anchorage." It should be noted, however, that the f'_c (ultimate unit compressive strength) is determined upon a different basis than has been usual in the past, and a great deal of what most of us have been calling 2,000-lb. or 2,200-lb. concrete would be 1,500-lb. concrete in this specification. Some of the other allowable stresses have been increased in this specification, and whether its adoption is going to mean that we use higher stresses in bending, for example, or lower stresses in bond depends upon whether we are going to improve the quality of our concrete or not. It is probable that we are, but when we consider all classes of commercial work in concrete it does not seem a certainty.

The "ordinary anchorage" mentioned as a condition for using the bond stresses already quoted is defined as follows:

In continuous or restrained beams negative tensile reinforcement shall extend full bond distance beyond the face of the support and this anchorage shall be not less than the depth of the beam. Where an end support has less width than ¾ the depth of the beam the rods to be anchored must be bent down on the outside face of the support a distance equal to the depth of the beam. Further, negative reinforcement shall be carried to or beyond the inflection point and ¼ of the positive steel shall extend 10 diameters into the support. In simple beams ¼ of the reinforcement shall extend 10 diameters into the support. These requirements seem to represent good practice and inflict no hardship on the designer.

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* Of Clifford & Roeblad, Consulting Engineers, Boston, Mass.

There follow special anchorage requirements to be met as a condition for using higher bond and shear stresses. These requirements call for ⅓ the positive steel to extend into the support to develop ⅓ the allowable stress, which is one-third more rods than the "ordinary anchorage" and an extension of a few diameters. For the negative steel ⅓ of it must be anchored for ⅓ the stress beyond the inflection point instead of just reaching it. For positive steel in simple beams ½ the steel must extend into the support to develop ⅓ of its stress; again a slight increase in steel over the ordinary requirements.

Having complied with these special requirements, higher bond stresses may be used provided the total tensile stress at a point of abrupt change in stress or a point of maximum stress does not exceed the value of F in the formula $F=Qu\Sigma oy+u\Sigma ox$, where "x" equals the length of the bar added for anchorage, including the hook, if any, "y" equals the distance from the point at which the stress is computed to the beginning of the anchorage, and "Q" is the ratio of the average to the maximum bond stress in the distance "y". The other letters have the same meaning as in the ordinary bond formula.

This formula has a rather strange and complicated appearance at first glance and it is difficult to foresee all it might mean in continued drafting room use. There are certain features which stand out after a little study, however. The first is that the second term being the bond value of the anchorage, the equation would be satisfied with about 40 diameters anchorage (at the stresses most of us have been using) even if the first term were zero. The first term represents the bond value of the rod under stress reduced by the ratio Q. If the beam has a concentrated load at the center, the shear, hence the bond, is constant and Q will equal unity. In the more familiar case of uniform load, the shear and bond vary uniformly, the average bond is one-half the maximum and Q will equal one-half. The first term of the equation will therefore have a finite and ordinarily considerable value. Now this formula must be applied at points of maximum stress and points of abrupt change in stress. The points of maximum in the ordinary beam will of course be the center and the support. The points of abrupt change in stress will appárently be at bends. At the center of a beam, or wherever maximum positive moment occurs, the first term will have considerable value, which will reduce the anchorage required. At the support "y" would equal zero, so the first term would drop out and a full anchorage would be required. At points of bending the conditions would be between these cases in the ordinary beam. It would seem that the requirements for satisfying this part of the specification would not be excessive in material, but some ingenuity will apparently be needed to devise a way of applying this formula in ordinary routine work.

The startling feature of this portion of the report is that having satisfied these not excessive requirements there is no limit to the stress we may use in bond. Without knowing the data on which the committee worked it is not obvious why this should be so, even realizing that high bond stress comes usually in beams short and heavy and strong, as an arch or frame.

There is a paragraph on anchorage of web bars which requires that they shall be anchored by providing continuity with the longitudinal reinforcement, by bending around the longitudinal bars or by a semicircular hook having a radius not less than four times the diameter of the bar. It further provides that hooks shall engage sufficient concrete to prevent the bar from pulling out, which does not seem very definite or clear cut; and that the anchorage shall be sufficient to develop the stress in the stirrup at a distance of d/3 from either face. A footnote says this is satisfied by a properly anchored stirrup whose diameter does not exceed 1/50 the depth of the beam, which would seem to be not far from current practice.

The limiting radius of 4d for bends appears to be a general requirement, and it appears that a hook has no greater value in these specifications than the same length of straight bar, which seems somewhat severe.

With the exception of the points noted, particularly the lack of any limit to bond stress in beams with special anchorage, this section of the report seems to be clear and an improvement over current practice, with no requirements which could be considered a hardship to the users of concrete.

FLAT SLABS

(Covering Part F, Chapter XI of the Report)

By Mark Linenthal, Member, Boston Society of Civil Engineers*

To appreciate the recommendations of the Joint Committee in regard to flat slab design, it is well to bear in mind the conditions which prevailed prior to their adoption. When flat slabs, or mushroom slabs as they were then more commonly termed, were first built, they were proportioned as seemed best in the light of the experience and judgment of the individua designer. With little experience and no established standards to serve a a guide, the designs of different engineers to meet similar condition showed startling variations. The reasons advanced for these variation were often even more startling than the designs. An early step towar formulating a common basis for flat slab design was made by John R Nichols when he called attention to the statical limitations in the design of such slabs. Keeping those limitations in mind, and with a growing mass of data from tests and observation of flat slabs under load, the firs recommendations of the Joint Committee were formulated. They hav since served as the basis of most of the building ordinances throughou the country, and have, to a great extent, standardized the practice o designing engineers in this field.

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* Consulting Engineer, Boston, Mass.

The first recommendations of the Joint Committee recognized tha the actual stresses in the concrete and the steel of flat slabs were materially less than the stresses which would exist, as figured in the ordinary fashion, if the total statical moment were resisted. It was, therefore recommended that the structure be designed to resist a total moment o about 85% of the total statical moment.

The second important recommendation of the Joint Committee in 1916 referred to the distribution of the bending moment as between positive and negative moments and as between the middle portion of the slab and the column strips. It was recommended that about 5/8 of the moment be taken as negative and that the column strips be designed to carry a very large proportion of the moment as compared with the middle strips

Recommendations were also made defining the critical sections limiting the minimum thickness of slabs for floors and roofs; and rules were established for determining concrete stresses, minimum sizes of dropped panels and column heads, treatment of end bays, flat slabs resting on walls, etc.

When Professor Sutherland first spoke to me about studying the latest recommendations of the Joint Committee on flat slab design and reporting on them, I guessed that it would be a comparatively simple task to analyze them, note the differences from the old report, and then be able to say exactly how flat slabs were to be designed henceforth giving the reasons for any changes that there might be from present practice. I find, however, after considerable study not only by mysel but by Mr. Hardison and Mr. Horovitz of the Kalman Steel Company and after several discussions between us, that all I can do is to point to some of the outstanding features and possibly open the way for more discussion.

In comparing the latest report of the Joint Committee with the revious one, we find several notable changes. First, the total bending oment in one direction has been materially reduced. In the old report was set at 85% the total statical moment. The present report sets it t 72%.

In two-way reinforced slabs, the ratio of total positive to total egative moment and the distribution of moment as between column trips and middle strips has been modified very slightly, if at all.

For the four-way reinforced panel a notable change has been made in he distribution of positive moment. It has been made the same for olumn and middle strips; that is, it is assumed that the positive moment substantially uniform across the panel width.

In this connection it would be interesting to work out the proper dis-ribution for other arrangements of reinforcement, such, for instance, as ears Mr. Smolski's name.

The old report permitted a considerable variation to the designer in he distribution of bending moment. This possible variation has been aterially reduced in the new report. I anticipate that the larger llowable variation in the old report will be preferable when it comes to electing slab thicknesses and steel sizes to fit ordinary practice.

In the new report the formula for minimum slab thickness outside he dropped panel has not been changed, except that no account is taken change in the width of the dropped panel. The arbitrary ratios of ab thickness to column spacing, 1:32 for floors and 1:40 for roofs, remain nchanged.

A new formula is given for the determination of the minimum thick-ess of the dropped panel and for slabs without dropped panels. Its erivation is not obvious. If we assume that it is based on the negative oment of two column strips and for an end span, then we can equate the alue of slab thickness in the formula with that given in the commonly sed formula $M=Kbt^2$ and derive a value of K. If this is done, it would ppear that K varies from 85 to 126 as $c:l$ (ratio of column head diameter column spacing) varies from 0.1 to 0.25. These values of K correspond concrete stresses (if we assume the steel stress at 18,000 lbs. per sq. in.) 590 to 750 lbs. per sq. in. For ordinary concrete mixtures it would enerally give greater thicknesses than the old recommendation. It akes no difference as between strong concretes and weak concretes.

Where the width of the dropped panel is assumed as a given propor-on of l_1, the panel width; c as a given proportion of l, the panel length; en, for a given value of R, the ratio of negative moment in the column rip to total moment, we can reduce the formula for the thickness of the dropped panel, or for the slab where there is no dropped panel, to the form of $t_1 = cl\sqrt{w_1}$ (w_1 being the total load per unit of area). This is the shape in which the formula would probably be used in ordinary practice.

The provisions for the dimensions of dropped panels permit a minimum of 0.3, the column center dimensions, as against 0.4 in the old report.

In this connection it is notable that there is no requirement for an enlarged column capital. The absence of such a capital increases the thickness of the slab, but it is not contrary to the recommendations.

For wall bays the total increase of the moment is substantially unchanged from the old recommendations, but the increase is larger for the positive and less for the negative moment.

Provision is made for varying the bending moment when a beam or wall offers support on a column center line. The negative moment in the middle strip at the wall is increased 30%. The column strip adjacent to the beam or wall is designed for a moment one-half that of the typica panel.

A change has been made in the determination of shearing stresses Two critical sections for which the shear is to be figured have been defined

1. Around the column capital and at a distance of the effective depth of the dropped panel beyond the edge of the capital.

2. Around the dropped panel and at a distance of the effective depth of the slab beyond it.

The average allowable unit shear is made to vary with the proportion of the negative steel within the column strip which crosses entirely over the column capital or the dropped panel, as the case may be. With a variation in this proportion, the shearing stress may vary from 2½% of the ultimate strength of the concrete to 3%, as a maximum.

Formulas for stresses in the concrete are recommended, but they are not clear to me. I do not see their derivation. On account of their complexity, I cannot tell, in the time I have had for study, what they would lead to in practice.

In a number of ways the new recommendations show a marked advance over the recommendations of 1916. The method of handling the shearing stresses appears very much clearer, and the reasons are quite obvious. The decrease in the total moment is in line with various tests which have been made, and with our experience with flat slabs in use. A departure from the conservatism of the first report, due to the meagre data then available, is quite justifiable now. The elimination of the requirement of enlarged column head will, I think, lead in some cases to more desirable buildings. Where column heads interfere with piping hafting, etc., and make partitioning expensive, they can now be eliminated. It is done at some extra cost but not at the expense of safety, ince there are corresponding increases in the thickness of slab and its einforcing when the column heads are reduced. What variations have been recommended in moment distribution will, I think, appeal to most designers.

There are other matters in which I consider the new recommendaions to be capable of improvement. It is recommended that slabs without dropped panels be made as thick as the dropped panel would be if the lab with dropped panel were used. It would seem to me to be obvious hat a thickening in the middle of a slab would result in an increased positive bending with a corresponding reduction in the negative bending. If this is correct it would permit a slab without dropped panel to be hinner than the dropped panel, but thicker than the slab outside the dropped panel where the dropped panel is used.

There is a larger objection of a general nature which I would make o the new recommendations. They contain several formulas of a more or less complicated nature whose derivation is not given. No intimation is even made of the basis upon which they were derived. Complexity, although not a virtue, is no particular objection if it has a proper basis; but it is hardly fair to expect designing engineers of ordinary intelligence to accept such rules on faith alone. If there were no other reason, this objection would be valid because the designer does not know how to modify the formulas for special conditions, although such modifications are suggested in the report. Another general objection I have to offer is against the recommendation of the use of formulas for thickness of slab which show no particular relation to the ordinary formulas used in concrete design. It would seem to me to be preferable to specify spans, widths of bands, and stresses in the concrete and the steel from which to determine the thicknesses rather than to determine them by the use of formulas which have no apparent relation to the stresses. It may be that the matter of deflection is involved in some of these formulas. In that case it would seem to me to be more desirable to develop formulas for deflection which should not be exceeded, but to make this entirely independent of the matter of stress determination.

My objections, unquestionably, will be suitably met, either by an explanation of the recommendations objected to, or by proper changes. When that is done these latest recommendations of the Joint Committee in reference to flat slabs will mark a substantial step forward in this part of the field of structural design.

FOOTINGS AND RETAINING WALLS

(Covering Parts H and I, Chapter XI of the Report)

By B. A. Rich, Member, Boston Society of Civil Engineers*

Part H — Footings

The specifications for the design of footings, sections 172 to 183 inclusive, are for the most part based on data obtained from tests made under the direction of Prof. Arthur N. Talbot in 1908 to 1912. These tests have been recorded and carefully analysed in Bulletin No. 67 of the University of Illinois Experiment Station. The methods for designing footings deduced therein are given in the more recent textbooks and are probably in quite general use. This subject has not been included in previous reports of the Committee.

The most notable change from the methods given in recent textbooks is the omission of any reference to punching shear. Punching shear has been used quite commonly for the determining of the depth of the footing at the edge of the supported member.

The methods outlined in these specifications relate to footings with two-way reinforcement only. The experiments by Professor Talbot on footings reinforced in four ways did not cover a sufficient range to warrant definite conclusions. The great objection to four-way reinforcement is that the four layers of bars at the intersection cause a marked difference in the location of the centers of the steel reinforcement. A circular shaped footing could be figured in a manner similar to that used for square footings, by dividing the circular base into quarters and proceeding as in the design of a square footing. The width of the assumed beam should be reduced somewhat from that assumed for a square footing.

In the first paragraph of section 176 it will be noticed that a distinction is made between footings with concrete columns bearing directly on them and footings having a steel plate or casting bearing on them. In the case of the concrete column there is no doubt that some of the horizontal compressive stress is transferred to the column by friction and bond. In the case of the metallic base there is little if any so transferred. The moment at the middle of the base according to these specifications, using the symbols given later in this section, is $M = \frac{2w}{3}\left(\frac{a}{2}+c\right)^2 c$, which is larger than the moment at the face of the column, *i.e.*, $M = \frac{w}{2}(a+1.2c)\,c^2$.

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* Of Rich, Bigelow & Tirrell, Consulting Engineers, Boston, Mass.

The assumption of a square having an area equivalent to the area of a circular column seems proper.

The arrangement of the bars outlined in section 177 agrees with the results obtained from the experiments mentioned above. However, no footing tested conformed fully to the scheme outlined, but a comparison of tests 1822 and 1823, and 1833 to 1838 inclusive shows that the grouping of the bars near the column is the most efficient arrangement. All these tests had the same number and size of bars and varied only in the arrangement. The highest results were obtained in tests 1822 and 1823, and 1837 and 1838. The first pair had no bars under the column, the bars being evenly spaced outside the column area. The second pair had all the bars confined to a space 12 inches either side of the center of the footing. The other tests had all the bars confined to belts of different widths near the edges of the footings. The average ultimate load for 1822 and 1823 was 204,000 lbs. and for 1837 and 1838 220,000 lbs. The pair with no bars within 18 inches of the center of the footing averaged only 133,000 lbs.

Section 178 specifies that "the extreme stress in sloped or stepped footings shall be based on the exact shape of the section for a width not greater than that assumed effective for reinforcement." The depth required to resist the diagonal tension or shearing stresses is generally such that the compression in the concrete is well below the working stresses permitted.

The diagonal tension or shearing stresses as determined under section 180 (which refers the reader to sections 132 to 134 inclusive) are the controlling factors in the design of a footing. The direct computation of the depth is a cumbersome operation at the best. Probably the quickest way is to make a few trials with different values for "d", the depth. If the top of the footing is to be sloped, a depth can be assumed at the edge of the column and the necessary depth computed at the edge of the perimeter of the lower base of the frustum of a pyramid or cone as described in section 133. Diagrams or tables would be of great help in the solution of this part of the design.

Designs can be made conforming to these specifications which would have punching shearing stresses of 400 lbs. or more per sq. in. The tests by Professor Talbot give punching shearing stresses of over 600 lbs. per sq. in. without failure. Tests made to obtain as near as possible to true punching shear have given ultimate stresses of over 50% of the compressive strength.

Bond stresses, section 181 (referring to sections 135 to 141 inclusive) are quite complicated. First, the bond stress is to be computed at the face of the column or bearing plate, *i.e.*, the point of maximum shear. Then the point at which the tension in the concrete is 40 lbs. per sq. in. is to be obtained. The average bond stress between these two points is to be worked out and a ratio "Q" of this average to the maximum bond stress determined. No limit is placed on the permissible average bond stress. The anchorage beyond the point of 40 lbs. per sq. in. tension in the concrete must be designed according to section 140b, *i.e.*, all bars are to be anchored for one-third of the maximum working stress in the steel. With the above data the value of "F" in formula (35) can be determined. If this is less than the assumed working stress in the steel the anchorage length, "x", can be lengthened to take care of it. The permissible bond stress in footings, according to section 138, must be 75% of those given in sections 136 and 137, as they have two-way reinforcement.

The transfer of the compressive stress in the longitudinal reinforcement in the heavily reinforced columns to the footings can best be accomplished by the use of distributing bases. The depth of embedment for the dowels for such heavily reinforced columns would have to be much more than the depth required to take care of the diagonal tension.

The permissible pressure on the top of a footing as determined by formula (49) can run to very high values. For example, suppose we had a footing 8 feet square with a column 1 foot square bearing on it. The above formula gives for the allowable bearing pressure on the top of the footing

$$r_a = 0.25f'_c \sqrt[3]{\frac{A}{A_1}} = 0.25f'_c \sqrt[3]{\frac{64}{1}} = 0.25f'_c \times 4 = 1.00f'_c$$

which is the ultimate compressive strength of the concrete. It seems evident that some limit should be placed on this value.

The specifications for pedestals in section 183 correspond with the common practice as far as the vertical-sided pedestals are concerned. The limitations imposed by the second paragraph on stepped or sloped pedestals, that "the depth . . . shall not exceed three (3) times the least diameter of the section midway between the top and bottom" makes the side slopes steeper than the common practice of one horizontal to two vertical. For example, if we assume a square pedestal with sloped sides, the top diameter being "a" and the offset to the edge of the base being "c", the diameter of the base will be a+2c and the diameter of the midsection will be $\frac{2a+2c}{2} = a+c$. Now the maximum value of the depth, "d", is 3 (a+c) = 3a+3c. Since 3a has some value 3c must be less than "d", or, in other words, "c", the offset, must be a rate of less than 1 horizontal to 3 vertical.

PART I — REINFORCED CONCRETE RETAINING WALLS

Reinforced concrete retaining walls have not been included in the previous reports of the Committee. The requirements of this part of the specifications need little comment.

In section 185c no mention is made of the fact that the loads from the slabs are transferred to the counterforts by tension. It would seem, therefore, that the compressive stresses at the supports should be not over $0.40f'_c$.

In section 185h no mention is made of distributing the temperature and shrinkage reinforcement over the surface.

J. R. WORCESTER, MEMBER, BOSTON SOCIETY OF CIVIL ENGINEERS*

(By letter)

THE new Joint Committee Report is of great importance to designers of reinforced concrete, and when generally accepted, as it undoubtedly will be, will have a marked effect upon designs. It is interesting to note that the changes from the 1916 report without exception — so far as the writer has observed — are all in the line of more economical construction. This indicates that the earlier report, where it erred, leaned towards conservatism. There is no question that much information has been acquired by experiments and by experience during the past eight years, and this compilation, which embodies much of this newly gained knowledge, should effect widespread economies.

Notable among the changes in design methods is the increase in the compressive fiber stress in concrete in beams, which will allow the percentage of steel in a balanced beam of 2,200-lb. concrete to run up to nearly 1¼. Another is the increase in shearing stress, with fully reinforced beams to $0.12f'_c$. Another is the allowance of a portion of tile blocks as carrying shear in combined tile and concrete construction.

While not being qualified to criticise provisions in the report which are based upon research, the writer desires to question three features which seem to be matters of opinion.

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* Consulting Engineer, Boston, Mass.

In section 165, which gives the formula for the design of longitudinally reinforced columns, it appears that the entire area of the concrete is allowed as taking stress. It seems as if the requirement of the former report and of our Boston law that a certain fireproofing shell should be neglected in the computation of stress is a wise provision which it would be well to retain.

The formula in section 168, for columns consisting of a structural core surrounded by concrete, is hardly consistent with the requirements for structural steel columns. Neither does it seem consistent to so disregard the value of "n". The application of a column reduction formula to the core in this case, when in longitudinally reinforced columns such reduction is not made, is again inconsistent. Would it not be more proper to use for the stress in the steel the flat unit for structural columns of moderate slenderness, and then to apply a reduction formula such as (47) to the column as a whole?

The writer is disposed to question the increase in unit stress, in section 194, allowed on intermediate grade, hard grade and rail-steel bars. The modulus of elasticity in these is no greater than in structural grade steel and the metal is not so reliable. The advisability of using 18,000-lb. in reinforcement is questionable, but, if allowed at all, it should be in structural grade billet steel.

Of General Interest

TO CO-OPERATE WITH PLANNING BOARD

Advisory Committee on Public Improvements Appointed

An important feature of the work of the Boston City Planning Board has been its success in enlisting the co-operation of the various civic organizations on matters of municipal interest. The enactment of the zoning law for Boston last year serves as a prominent example of the progress which can be made by such united effort. The latest development along these lines has been the appointment by Mayor James M. Curley of an Advisory Committee on Public Improvements to work in co-operation with the City Planning Board. This committee is made up of one representative each from fourteen Boston organizations. The members of this committee, selected by the mayor from nominations made by the different societies, are as follows:

Dugald C. Jackson, Affiliated Technical Societies of Boston; Francis P. Fenton, Boston Central Labor Union; Gifford LeClear, Boston Chamber of Commerce; Alton E. Briggs, Boston Fruit and Produce Exchange; Richards M. Bradley, Boston Real Estate Exchange; Ralph W. Gray, Boston Society of Architects; Arthur C. Comey, Boston Society of Landscape Architects; Elbridge R. Anderson, Massachusetts Real Estate Exchange; Leonard C. Wason, Master Builders' Association of Boston; Day Baker, Motor Truck Club of Massachusetts, Inc.; George W. Mitton, Retail Trade Board; George F. Stebbins, Team Owners' Association, Inc.; Raymond P. Delano, United Improvement Association; Mrs. Frederick T. Lord, Women's Municipal League of Boston.

Professor Dugald C. Jackson, who has been named to represent the Affiliated Technical Societies of Boston on this committee, is the head of the Department of Electrical Engineering at the Massachusetts Institute of Technology, and a member of the firm of Jackson & Moreland, consulting engineers, Boston. Professor Jackson has taken a prominent part in the activities of the Boston Chamber of Commerce as a member of its committees on public utilities and on municipal and metropolitan affairs and is unusually well qualified to serve on a committee of this character.

The Boston Society of Civil Engineers has, through its representatives, taken an active part in much of the work of the City Planning Board. Throughout the study of zoning for Boston a special Zoning Advisory Commission, made up of representatives from a similar group of organizations, co-operated with the Planning Board. Professor C. Frank Allen represented the Society on this committee and did a great deal of constructive work. Later when the zoning bill was before the Legislature the Council of the Affiliated Technical Societies of Boston endorsed the bill and appeared in its favor before the legislative committee. At the present time Mr. F. O. Whitney, treasurer of the Society, is the representative of the Boston Society of Civil Engineers on the Board of Zoning Adjustment.

Mr. Frederic H. Fay, of Fay, Spofford & Thorndike, consulting engineers, Boston, and a past president of the Society, is the present chairman of the Boston City Planning Board.

The new Advisory Committee on Public Improvements will not only aid the City Planning Board directly in its studies but the co-operation of the societies represented will help in furthering projects which receive the committee's approval. In this work the engineers can be of particular assistance due to their training and experience. This résumé of the activities of the Boston Society of Civil Engineers and of the Affiliated Technical Societies of Boston in connection with the Planning Board shows briefly what has been accomplished, and points out the opportunities which the engineers have to be of further service in the civic betterment of Boston.

Three Useful Publications

How much time and effort are sometimes wasted because we do not know the whereabouts of the information of which we are in need. Our government departments accumulate and publish much valuable information which may at times be of the greatest service to us, while we, not knowing of its existence, either laboriously duplicate portions of it at great expense, or else go without it and accept an inferior result.

If one were confronted with an engineering project affecting a large area and which demanded first of all an accurate survey, where could he find any information which would help him to secure accurate results? Or, if he needed to determine accurately some very long distances, are there any distances already measured which will save time and expense? Are there any points already located? There surely are — thousands of them — in Massachusetts and vicinity alone; points determined with great precision, carefully adjusted, tabulated and published in convenient form for reference.

The United States Coast and Geodetic Survey publication, entitled "Triangulation in Massachusetts,"* gives the latitude and longitude of many hundreds of points which have been established by triangulation. These three words are enough, perhaps, for most of us; we know that we have no use for latitude, longitude or triangulation, and so do not go further into the matter. But we can all use distances and true bearings, and these are an essential part of this information. The distances between these points are known with an accuracy that we should have difficulty in attaining. The distances are in meters but there is included a conversion table which makes it easy to change these distances to feet. The bearings are all referred to the true meridian at the point at which the bearing is given.

For many purposes we would prefer to use ordinary rectangular co-ordinates and not be bothered with converging meridians and latitudes and longitudes, all of which are necessary in the survey of a whole continent. But this does not mean that we must throw away all of the results of the government triangulation and begin over again. The publication entitled "Relation between Plane Co-ordinates and Geographic Positions"† shows how latitudes and longitudes may be readily converted into rectangular co-ordinates and furnishes the necessary tables for making this change. With three or four triangulation stations available within or near the area we are to survey we should have at once the

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[*] Special Pub. No. 76, U. S. C. & G. S. Supt. of Doc., Washington, D. C. (30 cents).

[†] Special Pub. No. 71, U. S. C. & G. S. Supt. of Doc., Washington, D.C. (10 cents).

x, y co-ordinates of these points, and the accurate distances between them to use for base lines and traverses, and the true bearings to use as a means of referring all directions to the true meridian.

The third publication is entitled "Use of Geodetic Control for City Surveys."* It shows the importance of this matter of controlling the accuracy of the surveys of large cities and describes the methods of procedure in making such surveys; it may therefore properly be grouped with the other two papers just mentioned.

As examples of surveys which have at different times been based upon such information we may mention at random the surveys of Boston, Springfield, and Winchester, the Wachusett Reservoir, the Metropolitan Park Reservations, the Charles River Basin, and the Boston Army Supply Base site. These are fairly large areas but there is no difficulty in finding points that will aid in surveying much smaller areas.

The Complex

A Coast and Geodetic Survey engineer was making observations in Texas for compass declination. He was at his instruments, and taking copious notes, when a rangy Texan walked up, examined the outfit, and inquired:

"Whattyer doing young felly?"

The engineer, new and enthusiastic in the coast and geodetic survey service, replied in detail:

"I am determining the latitude, longitude and declination of the compass and establishing a permanent survey monument for the benefit of local surveyors. The compass pointing goes through a small change in a period of years and I am determining the exact pointing at this date. I am observing on the sun, and by astronomy I am able to establish a true north and south line. Then I determine the magnetic pointing of the compass, and the angle between the two gives me what I'm looking for — the declination of the compass."

But the native wasn't convinced. He spat at the instruments, gave his big hat a vicious tug, and announced:

"Ye can't fool me, young felly; ye're looking fer oil."

(*From a recent number of* LIFE)

A Boiler Drum Built in a Gun Factory

From the November, 1924, number of the Stone & Webster JOURNAL

Much has been said about the boiler and turbine that will operate at 1,200 pounds' pressure and which is now being installed in the new steam-electric station that Stone & Webster, Inc., is building for The Edison Electric Illuminating Company of Boston.

The theory on which the design of the higher pressure steam unit is based is well established. The equipment to be used follows the best present designs, modified to care for the higher pressure. The designs have been fully proved by years of operation and it is expected that experience with high pressures will indicate only minor adjustments of design.

The building of some parts of such a boiler, however, required unusual methods. The building of the drum for the high-pressure unit, for instance, was entrusted by The Babcock & Wilcox Boiler Company to the Midvale Steel Company, the maker of coast defense guns and other heavy artillery. It was made from an open hearth hexagonal ingot of steel, seventy-eight inches in diameter, weighing 262,000

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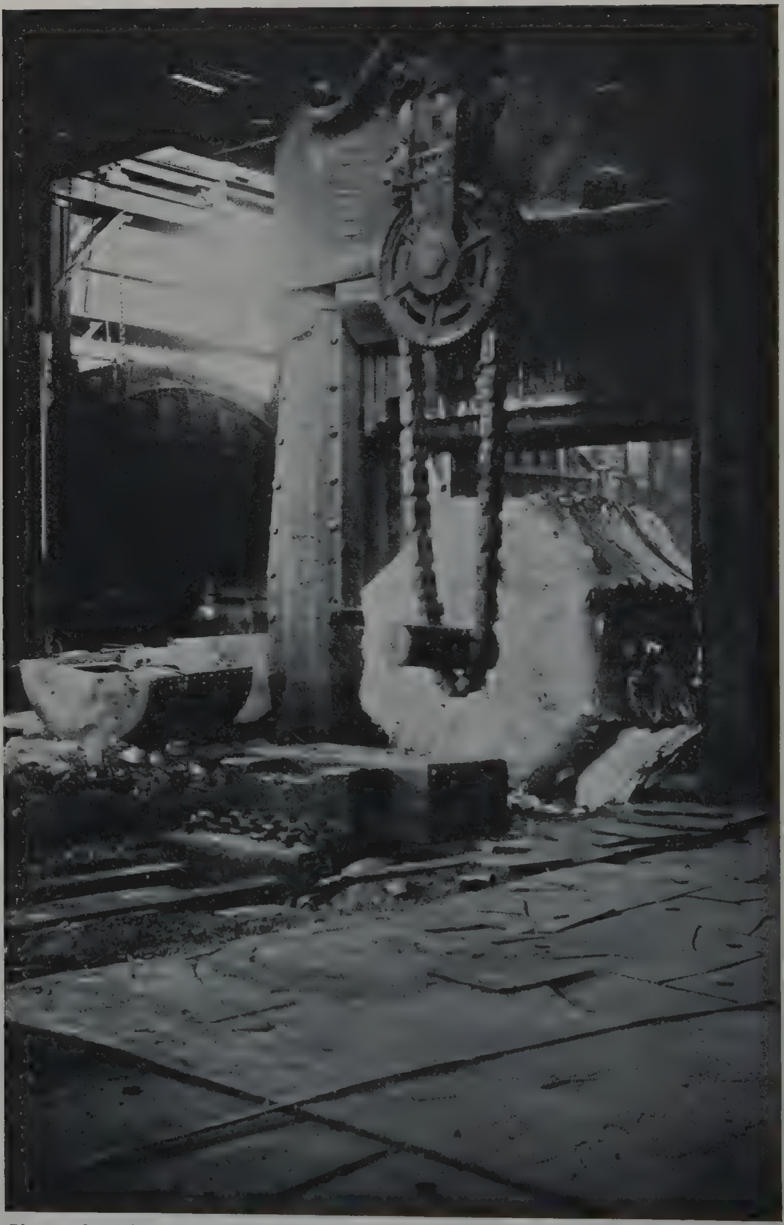
* Special Pub. No. 91, U. S. C. & G. S. Supt. of Doc., Washington, D.C. (20 cents).



Photograph furnished by the Midvale Company

THE FIRST STAGE IN MAKING A HIGH-PRESSURE BOILER DRUM

(From the Stone & Webster JOURNAL, November, 1924)

Photograph furnished by the Midvale Company

THE BOILER DRUM FOR THE HIGH-PRESSURE UNIT FOR THE WEYMOUTH STEAM-ELECTRIC STATION

(From the Stone & Webster JOURNAL, November, 1924) pounds, which, after passing through the various operations of being pierced, forged, annealed, machined and re-annealed above the critical temperature, emerged as the finished drum, thirty-four feet long over all, with an inside diameter of four feet and with walls four and one-fourth inches thick in the body and eight and one-half inches thick at the manholes.

Only one high-pressure unit is at present being installed along with the two 30,000 kilowatt units, and is so arranged that the latter can be operated with equal efficiency with or without the high-pressure unit.

Rainfall in Massachusetts in 1924

The excessive rains in the latter part of 1923 and in the winter and spring of 1924 raised the reservoirs depleted by the short drought of 1923 to their full capacity at the beginning of the summer, but during the remainder of the year the rainfall was on the whole less than the average, and a serious shortage again occurred in places using water in excess of the safe yield of their sources.

The average rainfall in the State as a whole in the year 1924 was 36.79 inches, or 7.75 inches below the normal as indicated by available records. There were rather wide variations, however, in different sections of the State, from a deficiency of some 4 inches in the Berkshire and 6 inches in the central and northern portions of the State, to more than 10 inches in the Connecticut valley sections. The areas receiving the most rain during the year were the eastern portion of Essex County, the southern part of Plymouth County and the western part of Barnstable County and a small area in the southern part of Worcester County, and the high lands of Berkshire County.

The seasonal distribution shows an excess of precipitation during the months of January, April, August and September, the greatest excess occurring in the latter month. Deficiencies have occurred in the other months, the most marked being in October, when very little rain fell in any portion of the State. The rainfall of August and September in the easterly portion of the State was about one third of the total precipitation of the year, and prevented what would otherwise have been serious shortages in some of the supplies in this region, but in October, November and December there was a deficiency of about 6¾ inches.

The rainfall on the Nashua River drainage area within the watershed of Wachusett Reservoir in the central part of the State was 6.68 inches below the normal and the average flow of the river was 1,035,000 gallons per square mile per day, or about 5.5 per cent less than the average flow for the last twenty-eight years. The flow of the Nashua River in twelve of the twenty-eight years since the observations were first begun has been less and often much less than its average flow in the year 1924.

Hydraulic Power Division of N. E. L. A. Meets

More than 300 engineers attended the Technical Sections of the National Electric Light Association at Cleveland, Ohio, February 4 to 6, 1925. The attention of the members of Boston Society of Civil Engineers is called particularly to the subjects under discussion by the Hydraulic Power Committee of the N. E. L. A.

These subjects included "Standardi zation of Methods of Rating Rivers" "Forecasting Water Supply", "Me chanical Reliability of Water Power Plants", "Fish Ladders", "Pitting o Hydraulic Turbine Runners", "Ex perience with Different Types of Dams", "Results of Draft Tube Tests", "Hydro-Electric Plant Layout", "Water Wheel Testing and Operating Records of Plant Discharge", "Penstock Design", "Vibration of Water Wheels".

During the course of the meeting many other important committees were in session. The work of the Hydraulic Power Committee has been referred to in detail because of its particular interest to civil engineers. The excellent reports and papers which are published by the N. E. L. A. form a valuable addition to the technical literature of the present day. The proceedings of the Technical Sections are available at the library of the Affiliated Technical Societies of Boston.

"L" Shops at Everett

At the October, 1924, meeting of the Designers Section, Mr. A. J. Blackburn, Architect, Maintenance Department, Boston Elevated Railway, gave a descriptive talk of the new car repair shops at Everett of the Boston Elevated Railway. Since that time there have appeared in the Electric Railway Journal three descriptive articles of these shops. These articles are well illustrated and it is believed that the men who heard Mr. Blackburn's talk may wish to refer to them. The articles are as follows:

1. November 22, 1924 — "Everett Shops of Boston 'L' Pave Way to Maintenance Economies". This issue describes the general preliminary planning and touches upon the congested conditions in the old shops existing previous to the construction of the new ones. General yard location plans are included, giving shop locations, trackage arrangement and other features.

2. December 13, 1924 — "New Boston 'L' Lumber Storage and Kiln". This issue describes the arrangement of the modern lumber storage building and kiln for the storage of car-finishing stock. Details are given of the duct construction, and interior and exterior photographs of the building are included. This article also contains a very good photograph of the transferway, which is a covered movable structure used for getting cars from the storage yards into the various shops of the car repair plant.

3. December 27, 1924 — "Boston 'L' Woodworking Shop Uses Individual Drive". This issue deals with the equipment units of the woodworking shop. Many photographs are included showing the machines in operation, dust collecting systems and other features of interest.

PROCEEDINGS

MINUTES OF MEETINGS

Boston Society of Civil Engineers

January 28, 1925. — A regular meeting of the Boston Society of Civil Engineers was held today in Chipman Hall, Tremont Temple, Boston, and was called to order by the President, Edwin H. Rogers, at 7.15 p.m. There were about 300 members and guests present.

The minutes of the previous meeting were approved as printed in the January Journal.

In the absence of the Secretary the President announced the names of those elected to membership by the Board of Government at its meeting today.

Members: Carl Travis Pinn, Everett E. Stone. *Juniors:* F. Tyler Carlton, Harold Glover Gamble, George Andrew Haskins, W. Douglas Lavers, Arthur Milliken Pillsbury, Charles Theodore Platter, Page Sanderson, Roger Gustavus Witherell.

Upon the recommendation of the Board of Government and on motion of Mr. F. E. Winsor duly seconded, it was VOTED that the regular February meeting be held on Friday, February 20, instead of on Wednesday, February 18, the regular day, in order that Mr. Robert Ridgway of New York City could address the meeting.

The President then introduced Colonel Claude H. Birdseye, Chief Topographic Engineer of the U.S. Geological Survey, who spoke on "Surveying the Grand Canyon of the Colorado." Colonel Birdseye was in charge of the expedition which made this survey in the summer of 1923, and his account of the trip was most interesting. His talk was illustrated with lantern slides and motion pictures. A vote of thanks was tendered Colonel Birdseye at the close of the talk.

Meeting adjourned at 9 P.M.

J. B. BABCOCK, *Secretary*.

DESIGNERS SECTION

JANUARY 14, 1925. — The regular January meeting of the Designers Section, of the Boston Society of Civil Engineers was called to order at 6.10 P.M. in the Affiliation Rooms. The reading of the minutes of the December meeting was omitted.

After a few remarks by Chairman Edward H. Cameron the meeting was turned over to Waldo F. Pike of the Boston Structural Steel Company and Clerk of the section. The general subject was "Structural Steel." Mr. Pike gave a brief outline of the process of fabricating steel and then divided the subject into the following parts: Design, Fabrication and Erection.

Mr. Wells of Densmore, Le Clear & Robbins, spoke on the subject of Design. Mr. John C. Moses of the Boston Bridge Works spoke on Fabrication.

The subject of Erection was to be taken up by Mr. Campbell of the Boston Bridge Works but he was detained out of town and unable to be present. Mr. Moses, upon the request of Mr. Pike, extended his remarks and touched upon the general features of Erection.

Mr. Whitecross of the Boston Building Department spoke upon the Boston building laws. An open discussion followed which brought out many points of interest. There were 108 members and visitors present. The meeting adjourned at about 8 P.M.

WALDO F. PIKE, *Clerk*.

SANITARY SECTION.

FEBRUARY 4, 1925. — A meeting of the Sanitary Section was held in the library of the Affiliated Technical Societies of Boston, being called to order by Chairman Gordon M. Fair at 7.15 P.M. In the absence of the Clerk, Mr. E. S. Chase read the minutes of the previous meeting.

Following an informal discussion on the subject of a place for a dinner preceding the annual meeting, the concensus of opinion was that the Chairman should make such arrangements therefor as he chose.

Mr. H. G. Baity, Assistant Engineer of the North Carolina State Board of Health, presented a paper on the "Control of Water Purification Works of North Carolina," discussing among other things the successful plan followed by his department in training filter plant operators. Following the paper there was a discussion participated in by Messrs. J. R. Pierce, Richard A. Hale and Almon L. Fales.

The attendance was 17. The meeting adjourned at 8.45.

H. P. EDDY, Jr., *Clerk*.

APPLICATIONS FOR MEMBERSHIP

[February 15, 1925.]

THE By-Laws provide that the Board of Government shall consider applications for membership with reference to the eligibility of each candidate for admission and shall determine the proper grade of membership to which he is entitled.

The Board must depend largely upon the members of the Society for the information which will enable it to arrive at a just conclusion. Every member is therefore urged to communicate promptly any facts in relation to the personal character or professional reputation and experience of the candidates which will assist the Board in its consideration. Communications relating to applicants are considered by the Board as strictly confidential.

The facts that applicants give the names of certain members as reference does not necessarily mean that such members endorse the candidate.

The Board of Government will not consider applications until the expiration of twenty (20) days from the date given.

For Admission

De Biasi, Charles Philip, Boston, Mass. (Age 21, b. Italy.) Is a junior at Northeastern University. On the coöperative plan has been with A. F. Sargent since September, 1923. Refers to H. B. Alvord, C. S. Ell, J. W. Ingalls, A. F. Sargent.

Ginsburg, Nathan, Boston Mass. (Age 20, b. Boston, Mass.) A graduate of Mass. Inst. of Tech. in 1924 with degree S.B. in civil engineering. Is now employed by the Boston Elevated as a rodman. Refers to J. B. Babcock, R. F. Connor, H. C. Hartwell, E. L. Lockman.

Harris, Harvey C., West Danville, Vt. (Age 36, b. Whiting, Kansas.) Studied construction and structural design timber, steel and concrete framing at Wentworth Institute two years. Experience: 1912–17, foreman, concrete construction; 1917–19, with 2d Engineers, overseas; 1921, June–November, foreman, concrete construction; July, 1924, to January, 1925, foreman, concrete construction, Rutland, Vt. Refers to B. A. Robinson, E. A. Varney.

Howe, John Augustine, Brighton, Mass. (Age 22, b. Brighton, Mass.) Graduated from Brighton High School, 1919, had one year's course at Lowell Institute, and one year's advanced building course at same school. Has been employed in order department by Carnegie Steel Company 1919–22; with Somers & Drisko, general contractors, 1922–23; New England Structural Steel Company, 1923, as steel detailer, also by C. B. Roberts Eng. Co.; and at present designer draftsman with Stone & Webster, Inc. Refers to W. W. Clifford, Dean Peabody, Jr., P. S. Rice, R. G. Stafford.

Klam, Joseph Elian, Cambridge, Mass. (Age 22, b. Syria) Graduate of Tufts College in civil engineering. Since graduating in 1924 has been employed by the Boston & Maine R. R. as structural draftsman. Refers to Robinson Abbott, H. C. Archibald, B. W. Guppy, Pusey Jones, N. S. Klink.

Nowell, Joseph Cornell, Jr., Cambridge, Mass. (Age 23, b. South Dartmouth, Mass.) Graduate of Mass. Inst. of Tech. with degree of S.B. in civil engineering. Since graduating has been in the employ of Stone & Webster, Inc., in the drafting division, where experience has been gained in detailing, designing and checking concrete work on a 60,000 Kw power house, and various other projects. Refers to L. S. Cowles, E. P. Rankin, R. G. Stafford, C. M. Spofford.

Noyes, William, Cambridge, Mass. (Age 28, b. Boston Mass.) Graduate of Harvard College, 1919, Harvard Engineering School, 1920. Experience as rodman with Northern Pacific, two months in summer of 1917; July, 1918, to November 1918, with the United States Naval Reserve Flying Corps; 1920, two months in the summer as assistant in surveying at Harvard Engineering School Camp at Squam Lake; 1921, February to October with Turner, Millspaugh & Safford, drafting and designing; November 1921, to February, 1922, engineer on construction with Holyoke Water Power Company; April, 1922, to October, 1923, with A. T. Safford, as resident engineer on construction; October, 1923, to December, 1924, with Howard M. Turner; since January 1, 1925, with Stone & Webster, Inc., as engineer in the hydraulic division. Refers to E. P. Hamilton, S. S. Kent, A. T. Safford, T. H. Safford, H. M. Turner.

Stimpson, Charles H., Jr., Weston, Mass. (Age 21, b. Weston, Mass.) Student at Northeastern University. Refers to H. B. Alvord, C. S. Ell, J. W. Ingalls, W. E. Nightingale.

NEW MEMBERS

Members

Herman Fay, Estimator, Allyn House, Hartford, Conn.
Carl T. Pinn, Transitman, 5 Gay Head St., Jamaica Plain, Mass.
Everett E. Stone, Member of Department of Public Utilities, State House, Boston.

Juniors

F. Vincent Britt, Student, 175 Larch Rd., Cambridge, Mass.
F. Tyler Carlton, Student, 316 Huntington Ave., Boston, Mass.
Henry B. Hilton, Student, 8 Franklin St., Danvers, Mass.
George H. Melcher, Student, 44 Buffum St., Salem, Mass.
Page Sanderson, Student, 6 Wiswell Circle, Wellesley, Mass.
George W. Sawin, Student, 9 Lombard St., Dorchester, Mass.
Roger G. Witherell, Student, 487 Tremont St., Taunton, Mass.
Carl A. Wolfrum, Student, 97 Bragdon St., Roxbury, Mass.

DEATHS

Walter E. Parker	October 26, 1924.
Willard Kent	December 22, 1924.
Charles H. Parker	January 28, 1925.
Frank P. McCarthy	February 4, 1925.

MEMOIRS

Frederic I. Winslow*

Frederic I. Winslow, who died on February 21, 1924, was the son of George W. and Jane Lucretia Winslow, and was born in New Bedford on January 30, 1863. He was of English descent, his American lineage dating back to 1630.

He was educated in the public schools of New Bedford and Boston; a subsequent course of four years in mechanical design at the South Boston School of Art, supplemented by private tutoring in mathematics, hydraulics and languages, well fitted him for his life work.

He spent thirty-six years of his engineering career in the employ of the city of Boston, entering the City Surveyor's office in 1881 and two years later the City Engineer's office as transitman in the Water Department. He quickly rose to the position of assistant engineer, and for many years was employed upon the varied problems of design and construction that present themselves in the maintenance and development of the water works system of a large city.

In 1916–18 he held the position of Assistant Supervising Engineer, United States Navy, in general research work and in the construction of the Destroyer Plant at Squantum, Mass.

In 1919 he entered the employ of the State of Massachusetts as division engineer in the Metropolitan Water and Sewerage Board, with headquarters at Framingham, Mass.; this position he held at the time of his death.

On January 15, 1892, he was married to Myrtle S. Smith, who with one son, Irving H. Winslow, survives him.

He was a member of the Boston Society of Civil Engineers (elected June 17, 1885), the American Society of Civil Engineers, the American Water Works Association, the New England Water Works Association and the Royal Arcanum.

He held the position of Librarian of the Boston Society of Civil Engineers for seven years, 1907–13, and that of Treasurer of the New England Water Works Association from 1922 until his death.

Mr. Winslow was, throughout his life, essentially a student, with an unusual power of reading rapidly and intelligently; his literary ability was marked and evidenced itself in numerous contributions to technical publications and magazines.

He was a kindly man of liberal thought and high ideals, who accomplished much that was worth while in his chosen profession.

He is mourned by a host of friends.

---

* Memoir prepared by Frank S. Hart and F. A. McInnes.

WALTER E. PARKER*

WALTER E. PARKER, who for many years was Agent for the Pacific Mills, Lawrence, Mass., and who was one of the foremost cotton manufacturers in the country, died on October 25, 1924.

Mr. Parker was born in Princeton, Mass., September 27, 1847. He received his early education in the public schools in Woonsocket, R. I. He entered the employ of the Social Mills at Woonsocket as an office boy, and worked his way up to the position of Superintendent, a position which he held for five years.

In 1881 he went to the Pacific Mills, Lawrence, Mass., as Superintendent of the Cotton Department. In 1887 he was made Agent of the Cotton and Worsted Departments, and in 1892 of the Print Works also. In 1909 the Cocheco Division at Dover, N. H., was also put under his charge. He retired from active service January 1, 1924. The position that the Pacific Mills holds in textile manufacturing is due largely to his guidance.

He was known internationally as an authority on the manufacture of cotton and worsted goods, and has been honored by many institutions in recognition of his ability in the textile industry.

He was interested in educational matters, serving for thirty years as trustee for Tufts College, and for shorter periods in other institutions.

He was a leader in local affairs, serving as president or director of many organizations.

He was most painstaking in the study of any problem, and when he had finished he had mastered it.

While his major work was in the manufacturing of goods, he was deeply interested in all of the problems of engineering which are connected with large undertakings.

His interest in engineering and scientific subjects is indicated by his membership in the Boston Society of Civil Engineers, which he joined in 1904. He was a life member of the American Society of Mechanical Engineers; member of the Royal Society of Arts of London, England; member of the Society of Arts of the Massachusetts Institute of Technology, and of the National Association of Cotton Manufacturers, of which he was a Past President.

Mr. Parker was a man of sterling integrity, a man of determination and perseverance, and of strong will, but with all he had a kindly disposition, was extremely patient, and was a man of few words. He had a characteristic possessed by few men who have held positions of great responsibility with many people working under them. He never said anything to a subordinate for which he would be sorry later, but every one knew whether he was satisfied or not with their performance. For this and because of other outstanding qualities he was a great and skillful leader.

CHARLES HAMILTON PARKER†

CHARLES HAMILTON PARKER, Superintendent of the Generating Department of the Edison Electric Illuminating Company of Boston, died at his home in Brookline, Mass., on January 28, 1925. His death comes as a great shock to his many friends and associates of the electric power industry of this country.

Mr. Parker was born at Hong Kong, China, on February 6, 1873. His father was the late Charles Edward Parker and his mother was Mary Phelps Parker. His father at that time was representing the oriental trading firm of Augustin Hurd & Company.

On the return of his father and mother to the United States, he entered the Massachusetts Institute of Technology, graduated in 1895, and entered the employ of the Edison Electric Illuminating Company of Boston. In a few years he rose to become Superintendent of the Generating Department, and his rare judgment in matters of company policy has always been valued by his associates.

During the Spanish War, Mr. Parker entered the Naval Brigade with the rank of lieutenant. He served on the U. S. S.

---

* Memoir prepared by Richard A. Hale and Charles T. Main.

† Memoir prepared by I. E. Moultrop.

"Catskill", which he brought from the Philadelphia Navy Yard to Gloucester, and later served on the "Marcellus" to the end of the war.

In 1906 he became lieutenant commander and in 1908 commander of the Naval Brigade. Three years later he was retired with the rank of captain.

During the World War he was made Chief of Bureau of the Massachusetts Public Safety Council and a member of the Naval Sub-base Committee. He was also given charge of all guard units on all Edison plants in Massachusetts.

In 1915 he married Helen Field Bidwell of Boston. He is survived by his wife, Helen F. Parker and his two brothers, James P. Parker of Jamaica Plain, Mass., and William Stanley Parker of Boston, Mass.

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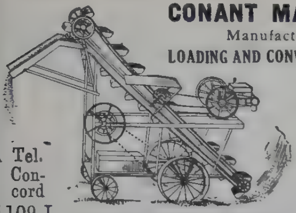

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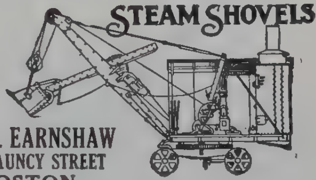

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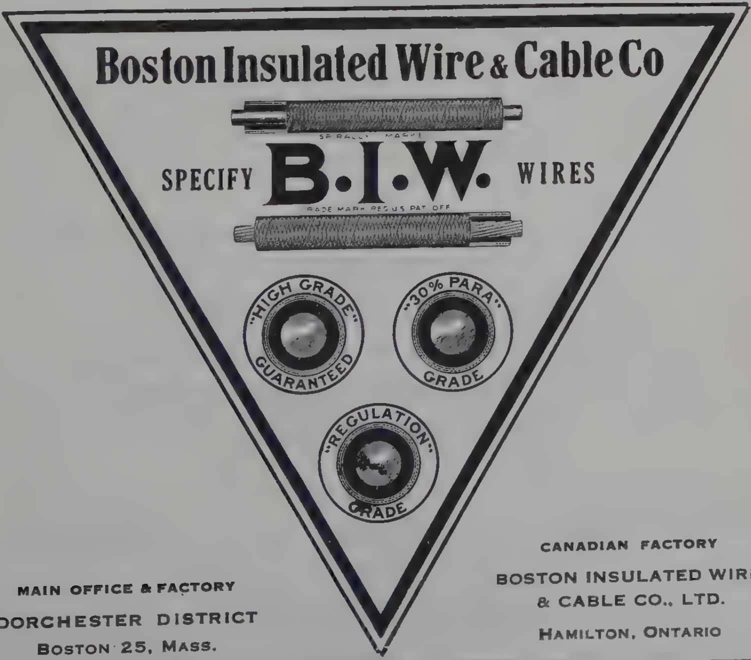

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